PERMEABILITY OF BITUMINOUS MIXTURES EVALUATION OF PAVEMENT DISTRESS

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ABSTRACT

The design of asphalt concrete paving mixtures requires a balance between different mixture properties. Some of these mixture properties include permanent deformation, fatigue cracking properties, strength, modulus and durability to name a few. There has been extensive work in trying to optimize these different mixture properties to produce a combination of aggregate and asphalt which will result in the maximum service life for the flexible pavement. Permeability has become a major concern in recent years among the asphalt paving community. It is important to have pavements that possess characteristics of low permeability which would minimize the effects due to moisture damage and increase the service life of pavements. The permeability of asphalt mixtures mainly depends on percent air voids, nominal maximum size of aggregate and the type of gradation. The infiltration of water in asphalt pavements promotes moisture damage primarily through damaging the binder cohesive bond and the adhesive bond between aggregates and binder. The first step in addressing the problems caused by the presence of water within pavement systems is quantifying the permeability of bituminous mixtures.

Initial Superpave implementation guidelines encouraged mix designers to develop coarse gradations for higher traffic level mixtures, as this was thought to produce a more robust aggregate structure. This notion to produce coarse aggregate bituminous mixtures was mainly adopted by countries like Pakistan where rutting is considered to be a principle form of pavement distress due to high temperatures and overloading prevailing on highway pavements. A problem that was experienced by National Highway Authority (NHA) of Pakistan during the rehabilitation of Nowshera-Peshawar project as part of National Highway Improvement Program was that of permeability of bituminous mix. The subsequent investigation showed that, the fine graded mixture performed slightly better than the coarse-graded mixture in terms of rutting resistance, fatigue cracking and permeability levels. This paper presents a description of pavement distress, asphalt mix design volumetrics, permeability testing and thereby improvements undertaken in the mix design to ensure a durable mix. This paper also intends to illustrate the experience gained relating to the permeability in the asphalt mix and drainage within the entire pavement system and its implication on durability of pavement as a whole.

1. INTRODUCTION

Resurfacing and Strengthening of 34 km of the National Highway (N5) between the cities of Nowshera and Peshawar north west of Islamabad was undertaken as part of the Highway Rehabilitation Project. The major works on this contract comprised removal of the existing asphalt pavement and its replacement with asphalt concrete base course and wearing course, augmented in some locations with an additional layer of aggregate base course. Physical works commenced on the site in December 2004 and at 31st March 2006 a length of approximately 24 km of asphalt pavement had been replaced and reopened to traffic.

2. SYNOPSIS OF DISTRESS

In January 2006, hairline cracking was observed in asphalt section 1 (4.2Km in length), the section of the completed pavement that had first been opened to traffic in June 2005. Cracking was then observed in the remaining sections subsequently completed with some isolated sections also exhibiting deformation and complete failure. Investigations immediately commenced after this cracking was observed with a series of inspection pits excavated to establish the conditions in each of the areas where the cracking occurred. Further investigations involving a review of all tests carried

out on the materials that were used for the works plus additional testing of the materials from samples taken from the works on site were carried out.

An initial review of the results of quality control tests that were done on the materials before and during construction confirmed that these were all within the specified limits in the contract indicating that probably materials problems were not the source of the failures. The nature of the cracking originally observed suggested what the possible causes of the failures might be and investigations proceeded in this direction. Concerns were however raised about the quality of the materials being used in the AC pavement layers and further testing was carried out on these materials to verify their suitability or otherwise. The areas of investigation included the following:

- Original pavement design
- Design strategy implemented
- Existing site conditions including the sub-pavement and embankment conditions
- Site Drainage Situation
- Materials used for the Works
- Job Mix Formula (JMF) Preparation for asphalt mixtures
- Pavement Investigations

The proceeding para's detail the regime and the results of the investigations that have been carried out to establish the reasons for the failures. Each of the above issues are examined further in this paper to establish their relevance or otherwise to the failures

3. INITIAL OBSERVATIONS

Cracking was initially observed in Section 1 after rains in January 2006. The fine hairline cracks, barely visible on a dry surface were observed as the pavement was drying out. The cracks were concentrated longitudinally along edge of the outer traffic lane in the outer wheel path, sometimes associated with transverse cracks. The entire area of AC pavement placed was then inspected with cracking reported to varying degrees in practically all areas. Some settlement was observed associated with the cracking, although not in all locations. In Sections 5, 6 and 7 the cracking was also associated with substantial deformation and complete failure of large areas where the aggregate base course layers appeared to be saturated with water. The cracking was observed in all sections irrespective of whether they had been completed since June 2005 (Sec 1) or January 2006 (Sec 7), with the sections subsequently completed (5, 6 & 7) showing overall less cracked areas, but more severe deformation and failures in some areas.

The diagram below indicates the timing of completion of the sections and when rainfall was recorded on site in relation to the initial observation of the cracks.

Section	Jun	July	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr
1											
2											
3											
4											
5											
6											
7											
Rainfall	Nil	Nil	Nil	Nil	Nil	Rain	Nil	Rain	Rain	Rain	

Cracking

4. INVESTIGATION METHODOLOGY

- A review of pavement rehabilitation design and drainage;
- A visual survey of cracking;
- Inspection Pits in and around cracked, uncracked and areas not yet touched by the contractor;
- Tests on aggregates and bitumen and tests on asphalt base course and wearing course from cracked and non-cracked areas including quality tests on aggregates from stockpiles, bitumen extraction from cracked and non-cracked areas, as well as from existing asphalt pavement;
- Cores through cracks to establish cracking pattern;
- "Porosity test", carried out on non-cracked, open to traffic asphalt wearing course;

➢ Falling Weight Deflectometer (FWD) tests over the entire length constructed for structural analysis of pavement structure.

4.1 Pavement Rehabilitation Design

The rehabilitation design of the pavement was based on AASHTO Design Guide of 1993 [1] where the provided structural number for the predicted design traffic of 10 years was satisfied. The rehabilitation comprised removal of the existing deteriorated asphalt layer of approx. 12cm and its replacement with new asphalt overlay of 16cm thickness, augmented in some locations with an additional layer of aggregate base course 10 to 15cm.The surface drainage of the pavement was also given due attention. It may be noted that the road under discussion was constructed in the early eighties on high embankment with an average height of approximately 1m. However, there are localized stretches were the Finished Road Level (FRL) is at level with the surrounding ground level. One of the sections i.e. section 7 which also failed was observed to qualify this scenario since a high canal embankment run parallel to the section and drainage in this section was under question. On other sections there seem to be no problem with disposal of surface drainage.

The drainage mechanism within the pavement structure was also analyzed and later investigated through destructive testing. It was established that since the existing pavement shoulders were constructed from subbase material with Plasticity Index (PI) = 0 i.e. non-plastic material, therefore drainage within the pavement structure was satisfactory. However, investigation into aggregate base course material within the existing pavement revealed some levels of plasticity and it was noted that the material was not free draining and was susceptible to entrap water for longer periods.

4.2 Visual Condition Survey

A visual inspection was carried out to record the various types of distresses which had appeared on the surface of the pavement. The distresses recorded included hairline cracks, alligator cracks, shoving, raveling, etc. It was also noticed that pot holes had started developing at the severely cracked areas. Some areas also showed signs of depression. There were no signs of bleeding or rutting on the pavement. The majority of the distresses were confined to the left hand wheel path of the slow lane. In the sections where the pavement had totally failed the distresses started from the slow lane and then progressed to the fast (inner) lane, which mostly cater for fast moving vehicles with low axle loadings. (see Figures 1 and 2 below): [2]

Figure 1: Intense cracking of the asphalt layer

Figure 2: cracking followed by raveling.

A number of inspection pits were dug, some in the centre of slow lane and others at the boundaries between the blacktop and the shoulders, down to the top of the sub-grade to check, among other things, the pavement layer thicknesses, the bond between the pavement layers, presence of trapped moisture between the layers of the pavement or the top of the sub-grade or evidence of weak foundation. Some of the pits were located in the distressed areas and others were at non-distressed areas. In all 17 pits were dug in the carriageway pavement and 14 were dug at the boundaries between the blacktop and the shoulders. The sizes of the pits were approximately 1m by 1m. The pits were carefully excavated layer by layer and logged. The layers were examined for thickness, bonding between each other, presence of water between the layers, segregation, for the asphalt layers coating of the aggregates with bitumen, and for the granular layers the moisture content of the material. [3]

From each of six sections (which have been opened to traffic asphalt concrete specimens were taken for extraction and gradation analysis. From each section, two samples were taken from the asphalt wearing course - one from a cracked area and the other from an un-cracked area. The same was done for the asphalt base course. General observations from the inspection pit are as follows:

- Higher moisture content in pits excavated in section 1; (see Figure 3 below)
- Subbase and base course of outer shoulders dry whilst adjacent base course is saturated, as is the subbase below the saturated aggregate base course;
- Free water between asphalt base course and asphalt wearing course layers and between asphalt base course and aggregate base course layers (this was observed during removal of cracked new asphalt layers as well as during sampling for bitumen extraction, both in cracked and uncracked areas; (see Figure 4 below)
- Increasing extent of area of cracking after rainfall;
- Dull appearance of AC samples removed for testing;
- Normal moisture content under pavement of untouched (old) sections and under outer shoulders;

Figure 3: High Moisture content within pavement.

Figure 4: Presence of water within pavement.

4.3 Core Samples

A total of fourteen (14) cores were taken, two from each of the distressed sections. From each section one core was taken from a cracked area and another from un-cracked area. The objective of taking cores from both cracked and un-cracked areas was to investigate whether bottom-up or top-down cracking had occurred. The observations made from the cores taken confirmed cracks initiated from the surface of the pavement and propagated downward through the asphalt layer. In some case the cracks were limited only to the top of the wearing course. Others have gone through the wearing course into the top of the asphaltic base course and yet in others the cracks have gone through both the asphaltic wearing and base courses. The observations confirm that "top-down" cracking had generally occurred. (see Figures 5 & 6 below):

Figure 5: Surface Crack on asphalt core.

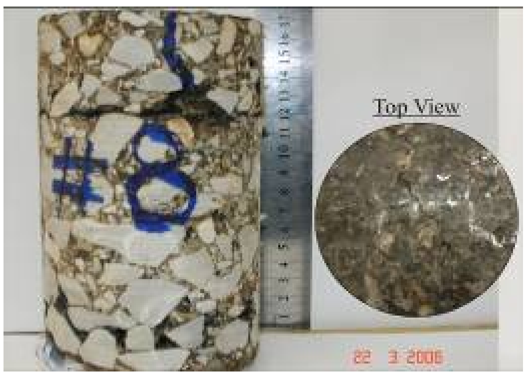

Figure 6: Crack propagation.

4.4 Porosity Testing

With the inception of the Superpave mix design the attendant problems with permeability of the asphalt mixtures have increased. There has subsequently been intensification in the development of equipment for field as well as laboratory permeability testing. Permeability is defined as "the rate at which a pressurized gas or liquid passes through a medium" or "the ability of a medium to permit flow". Most of the various permeameters in the market use Darcy's Law to determine the rate of water flow through asphalt pavements. [3]

Such equipment to determine permeability of the pavement layers was not available, but to obtain an indication of the degree of permeability of the AC layers in particular a simple test was performed. This test does not conform to a specific standard but provides an indication of the relative porosity of the layers.

The test consisted of pouring water on a confined area and timing the absorption of the water into the pavement. A 3inch sieve of 30.2 cm diameter was used as the confining device for the water (see Figures 7 and 8 below). This was placed on the pavement and sealed around its outer edges with silicone preventing the escape of water. Water was added up to a preset mark and after 20 minutes the level of water was recorded. The difference between the two corresponding volumes gives the volume of water that has penetrated into the pavement. Four permeability tests were carried out on four sections of the road, namely: On new asphalt concrete wearing course, on new asphalt concrete base course and on old asphalt concrete wearing course (2 tests at different locations). The following are the test results:

- ➢ The permeability of the new asphalt concrete wearing course was 0.475 cm/sec (20 minutes for a drop in level of 0.4 cm)
- ➢ The permeability of the new asphalt concrete base course was 6.48 cm/sec (2 minutes for a drop in level of 0.4 cm)
- ➢ The permeability of existing asphalt concrete wearing course was 0.00 cm/sec (for both tests there was no variation in the water level after 30 mins.)

The degree of porosity of the new mixtures is higher than might have been expected. Whilst the existing asphalt is virtually impermeable (as should be the case since it was an old aged material) with a wearing course, the new mixtures were very porous. That explains why the soils in the pits dug in the existing asphalt were dry whilst those in some of the pits in the new pavement were wet.

4.5 Falling Weight Deflectometer (FWD) Testing

FWD testing was carried out on the outer lane of the distressed sections at 100m c/c for a detailed back-analysis procedure to determine effective stiffnesses of the pavement layers. The pavement was considered to be fully flexible. The pavement structure was modelled as three layered structure (bituminous + unbound granular layer + subgrade). The aggregate base course and subbase material has been considered as a one single layer to extrapolate the effective stiffness modulus for the unbound material. The combination of both the unbound layers into one layer is attributed to the fact that aggregate base and granular subbase modelled separately may give variable results to extrapolate realistic moduli values.

Since the evaluation of the pavement structure of the respective sections was considered to be critical with regard to the existing strength and subsequent recommendation, therefore, it was pertinent to analyse the deflection data on different back-calculation software programs to account for any variability among the strength values of the pavement layers. For this purpose three (03) different software's were used namely; MODULUS, ERIDA and MICBACK programs. The values obtained

from each analysis for each pavement layer were checked and subsequently an average modulus value for the respective layer is reported in Table 1: [4]

Sections	Percentiles	Asphalt Layer Modulus (ksi)	Granular Layer Modulus (ksi)	Subgrade Modulus (ksi)
Section # 1	50^{th}	375	25	17
	85^{th}	282	16	15
Section #2	50^{th}	488	61	21
	85^{th}	375	26	18
Section #3	50^{th}	519	29	17
	85^{th}	259	15	14
Section #4	50^{th}	243	16	22
	85^{th}	208	13	20
Section #5	50^{th}	186	14	20
	85^{th}	142	11	18
Section #6	50^{th}	172	13	20
	85^{th}	135	10	17
Section #7	50^{th}	236	16	19
	85^{th}	150	11	16

Table 1: Averaged Back-Analysed Effective Layer Stiffnesses Resulting from Various Back calculation Software's (50^{th} & 85^{th} Percentile Values)

Visual and structural assessment of the distressed sections showed that the distress observed at site is of varying degree involving asphalt concrete and unbound granular material layers in most of the sections. The asphalt concrete layer in four sections (04) namely 4, 5, 6, and 7 had fully deteriorated and structurally could be termed as poor integrity material which cannot be used as a structural layer to support any further asphalt overlays or resurfacing. The results revealed that the unbound material in these sections is week due to high moisture penetration from deteriorated sealant layer and will need removal/reworking and relaying to restore its structural integrity.

The structural analysis show that the asphalt concrete in section 1 has deteriorated with weak underlying granular material (due to high moisture). Comparatively, section 2 is noted to be intact and in good condition. Minor surface cracks were present but of localised nature. The asphalt layer and the unbound material had reasonable strength values. The section needed nominal reinforcement in terms of an overlay to cover deficiencies in the asphalt mix. Section 3 was noted to be generally in fair to good condition. Visual and structural assessment also showed that the section needs partial removal of asphalt material and unbound material in weak stretches.

The subgrade strength as quantified provides a reasonable support for overlays in almost all the respective sections evaluated.

5. CONFIRMATORY TESTING OF MATERIALS

5.1 Aggregate Base Course

Aggregates are required to conform to certain specification requirements as stipulated in NHA General Specifications of 1998 [5]. Aggregates pertain to crushed materials with at least two mechanically fractured faces on 90% of the material retained on #4 sieve. Tests conducted to confirm the specification requirements include Los Angeles Abrasion, Soundness, Sand Equivalent, Flakiness and California Bearing Ratio (CBR). Aggregates classified as 2inch (50mm) were supplied from the Basay Baba quarry near Bara, approximately 25 km west of construction site. These aggregates were provided from several small crushers that operated in that area although the source of the rock is common to all. This results in variability in the gradations that was identified in the initial acceptance testing and that has been monitored during placement of ABC.

The results of the various tests conducted on the above aggregates and the specification requirements are shown in Table 2 below. It can be observed that all the test results conform to the specification requirements.

Description	Requirement	Test Results
Los Angeles Abrasion	40% (max)	27.1%

Sulphate Soundness	12% (max)	3.6%
Sand Equivalent	45 (min)	52.7%
Liquid Limit	25 (max)	Nil
Plasticity Index	6 (max)	NP
Particle Shape	15% (max)	5%
CBR	80 (min)	122%

Table 2: Aggregate Base Course Properties

5.2 Aggregates for Asphalt Concrete (AC) - Coarse Aggregates

Coarse aggregates are required to conform to NHA specification requirements. Coarse aggregates should be crushed rock, gravel or bolder with at least two mechanically fractured faces on 95% of the material retained on #4 sieve. Aggregates passing #4 sieve should be 100% crushed from rock or boulder. Tests conducted to confirm the specification requirements include Los Angeles Abrasion, Soundness, Sand Equivalent, Flakiness and Elongation Indices, Stripping, Affinity with bitumen and Petrography analysis.

Three sizes of coarse aggregate were used to achieve the required combined gradation for the JMF for asphalt base course These aggregates classified as 1½” (19-38mm), 1” (10-25mm); coarse and ⅜” (5-10mm) were supplied from the Basay Baba quarry near Bara, approximately 25 km west of construction site. The smaller two sizes were used for the combined gradation of JMF for asphalt wearing course. These aggregates were provided from several small crushers that operate in that area although the source of the rock is common to all. The results of the various tests conducted on the above aggregates and the specification requirements are shown in Table 3 below. It can be observed that all the test results conform to the specification requirements.

Description	Requirement		Test Results
	ACBC	ACWC	
Los Angeles Abrasion	40% (max)	30% (max)	26.2%
Sulphate Soundness	12% (max)	12% (max)	1.6%
Sand Equivalent	45 (min)	45 (min)	71
Flakiness Index	15% (max)	10% (max)	6.8%
Elongation Index	15% (max)	10% (max)	5.5%
Stripping	80% (min)	80% (min)	>95%
Affinity for Asphalt	Cationic/Anionic		Good
Petrography	Hydrophobic/Hydrophilic.		Hydrophobic (95%)
Water Absorption (10-25mm)	-		0.53%
Water Absorption (5-10mm)	-		0.46%

Table 3: Coarse Aggregate Specifications for Asphalt Mix and Test Results

Fine Aggregate

One size of fine aggregate was used to achieve the required combined gradation for the JMF. This aggregates classified as stone dust or “khaka” (0-5mm) was supplied from Margalla Quarry near Taxila, approximately 100km km east of construction site. Before the material was approved Atterberg’s limits and water absorption tests were conducted. It can be seen from Table 4 below that all the specification requirements are satisfied:

Description	Requirement		Test Results
	ACBC	ACWC	
Liquid Limit	25% (max)	25% (max)	Nil
Plastic Limit	-	-	Nil
Plasticity Index	6% max	4% max	Non-plastic
Water Absorption	-	-	1.59%

Table 4: Fine Aggregate Specifications for Asphalt Mix

Absorption tests were carried out on this material prior to its incorporation into the JMF. The results recorded in the JMF were between 1.42% and 1.73% with an average indicated of 1.59%. No specific range for this parameter is indicated in the specifications.

5.3 Bitumen

The binder specified for the Contract was 60-70 Penetration-Graded bitumen. A sample of the bitumen was sent to the Pakistan Council of Scientific and Industrial Research (PCSIR), Lahore for testing. Table 5 below shows the test results from PCSIR as well as the Specification Limits as per AASHTO M 20-70 (2000):

Description	**AASHTO Requirements**		**Test Results**
	Min	Max	
Penetration at 25°C, 100 gm, 5 sec	60	70	65
Flash Point, Cleveland Open Cup, °C	232	-	315
Ductility at 25°C, 5cm/min, cm	100	-	115
Solubility in Trichloroethelyene, percent	99	-	99.6
Thin-film Oven test, 3.2 mm, 163°C, 5 hrs (Loss on heating, percent)	-	0.8	0.3
Penetration of residue at 25°C, percent of original	54	-	55.38
Ductility of residue at 25°C, 5 cm/min, cm	50	-	52
Specific Gravity of Asphalt Cement			1.029

Table 5: Bitumen Specifications and Test Results

6. MIX DESIGNS
6.1 MIX DESIGN APPROACH

Currently it appears that al least two different methods are being used on NHA projects: [3]

a) Method 1, initially used by the Contractors on Highway Rehabilitation Projects followed a combination of MS-2 and a recipe approach. In the determination of the optimum binder content, the samples are prepared and tested as in the Marshal Method of Mix Design Graphs are then prepared for:

- Stability vs. Asphalt Content
- Flow vs. Asphalt Content
- Unit Weight of Total Mix vs. Asphalt Content
- Percent Air Voids (Va) vs. Asphalt Content
- Percent Voids in Mineral Aggregate (VMA) vs. Asphalt Content

The graphs of three parameters are then used for the determination of the asphalt content of the mix. The optimum asphalt content is the average of the asphalt contents corresponding to:

i) Maximum Unit weight of total mix
ii) Maximum Stability and
iii) Medium Air Voids.

b) Method 2 follows the Contract General Specification, which requires the designer to follow the Marshall Method in Asphalt Institute Manual Series No. 2 (MS-2), 6th edition or latest. The design (optimum) binder content is then determined as satisfying the following criteria as laid down in the General Specifications, viz

- Stability
- Percent Air Voids
- Flow and
- Percent Void in Mineral Aggregate.

This approach is more representative of the requirements and intent of the General Specifications. A third approach that could be considered would be similar to the Method 2 above except that the design binder selected is the binder content satisfying:

- Air Voids
- Voids in Mineral Aggregate
- Flow
- Stability and

➢ Voids Filled With Asphalt

This approach would follow the limits for these parameters set in MS-2 and not those stated in the NHA General Specifications.

6.2 Asphalt Base Course

The properties of this JMF are shown in Table 6 below:

Description	Specification Limits	Lab Trial	Plant Confirmation
% Voids	4 – 6	5.6	6.0
% VMA (Voids in Mineral Aggregate)	12 (min)*	12.9	13.5
% VFA (Voids filled with Asphalt)	--	56.6	55.6
% Stability (kg)	1200 (min)	1355	1370
Loss of Stability % at 60° C after 24 hrs	25% (max)	13.2	13.2
Flow (0.25mm)	8 – 14	9.4	10.2
Gmb (Bulk Sp Gr of Mix gm/cc)	--	2.383	2.367
Gmm (Max Sp Gr of Mix)	--	2.525	2.518
Gsb (Combined Bulk Sp Gr)	--	2.645	2.645
Gse (Effective Sp Gr)	--	2.657	2.657
Gb (Sp Gr of Bitumen grade 60-70)	--	1.029	1.029
Optimum Asphalt Content	--	3.3%	

* From Table 5.3, MS-2

Table 6: JMF for ACBC

6.3 AC Wearing Course

The properties of this JMF are shown in Table 7 below:

Description	Specification Limits	Lab Trial	Plant Confirmation
% Voids	4 – 7	5.8	6.1
% VMA (Voids in Mineral Aggregate)	14 (min)*	14.0	14.3
% VFA (Voids filled with Asphalt)	--	56.6	57.3
% Stability (kg)	1000 (min)	1219	1343
Loss of Stability % at 60° C after 24 hrs	20% (max)	7.7	12.2
Flow (0.25mm)	8 – 14	9.7	10.3
Gmb (Bulk Sp Gr of Mix gm/cc)	--	2.354	2.332
Gmm (Max Sp Gr of Mix)	--	2.500	2.521
Gsb (Combined Bulk Sp Gr)	--	2.633	--
Gse (Effective Sp Gr)	--	2.651	--
Gb (Sp Gr of Bitumen grade 60-70)	--	1.029	1.029
Optimum Asphalt Content	--	3.83%	

* From Table 5.3, MS-2

Table 7: JMF for ACWC

7. DISCUSSION

7.1 Asphalt Mix Design Methodology

Many countries base their asphalt mix design on Asphalt Institute Manual Series No. 2 (MS-2) whilst others, e.g. the United Kingdom use a recipe approach, similar to the NHA approach. The fifth edition of MS-2 issued in 1993 brought in a few revisions from the previous editions that remain valid in the 6th edition. This included the addition of a Voids Filled with Asphalt (VFA) criterion to Marshall mix design. One of the main purposes of introducing the VFA criterion was to make sure that the voids in the mineral aggregates are filled with enough asphalt to coat the aggregates and to provide both adhesion and cohesion in the mix and at the same time to provide enough voids in the mix for

secondary compaction and the expansion of the asphalt during hot seasons. In the table provided in MS-2, for heavy traffic and percent air voids of between 3 and 5, the percent Voids Filled with Asphalt is between 65 and 75. MS-2 indicates that anything less than the minimum is likely to end up in a lean mix and premature cracking and anything more than the maximum could end up in a rich mix resulting in bleeding and rutting due to plastic flow.

The NHA specification however does not provide any limits to VFA. It is therefore possible that a mix which satisfies all the NHA criteria will also satisfy the VFA criterion in MS-2, but at the same time another mix which satisfies all the NHA criteria may not satisfy the VFA criterion in MS-2. Although the mix approved for the asphalt wearing course on this rehabilitation project satisfied all the NHA criteria set out in the specification it does not satisfy the MS-2 VFA criterion; it is below the lower limit.

In the previous edition of the NHA General Specifications of March 1991 the limits for Voids Filled with Asphalt (VFA) were specified as 55-65 for ACBC and 65-75 for ACWC, but in the current NHA General Specifications issued in 1998 the limits for this parameter have been deleted. The NHA 1991 General Specification also provided a limit for VMA of 13 (min) for ACBC and 14 (min) for ACWC whereas in the updated 1998 General Specification the VMA limit is to be determined according to Table 5.2 in MS-2 Sixth Edition 1993.

It was concluded therefore that in deleting the limits for VFA from the NHA specification without making the MS-2 limits applicable as it did for VMA, NHA considered that there were local conditions prevailing that made the application of this parameter unacceptable. The problems that have been prevalent for some time in Pakistan with rutting of AC pavements due to the extremely heavy loads are likely to have influenced this decision such that lower bitumen contents were preferred with a result that the VFA limits indicated in the MS-2 table could not be achieved in practice. It has become increasingly evident that in using the coarser gradings specified it is very difficult to achieve a result that will also conform to the MS-2 requirements for VFA. The increase in binder content also appears to be contrary to the desire of NHA to have a relatively drier mix in order to minimize the likelihood of rutting developing.[3]

7.2 Asphalt Mix Design

Visual observation of asphalt concrete samples taken from some of the test pits suggests that the asphalt concrete was a lean mix. Asphalt concrete wearing course pieces appeared porous. There were signs of insufficient coating of the aggregates by the binder and some pieces could easily be broken by hand. This would normally be an indication that the binder content of the mix was on the low side of the design binder content. In this case however the design binder content satisfies the Specification criteria and the actual average binder content in production is as per the design.

It may be noted that the same binder content has been used for the asphalt mixtures on other contracts. This suggests that low binder content although a major contributing factor, alone is not the sole source of the problems. The larger nominal maximum aggregate size combined with the apparent lack of adequate binder content will make the mix porous and more permeable.

With the introduction of Superpave mix design in the United States in the middle of 1990, several states reported problems with greater than expected permeability associated with the use of coarse-graded mix, sometimes accompanied by less binder content. In addition, there was debate over the in-place air void contents and layer thickness needed to ensure an impermeable pavement. To overcome the problem with permeability, some states increased their field density requirements, lift thickness requirements or both. Other states opted to reduce the nominal maximum aggregate size of given lifts (e.g., use of a 19.0-mm in place of 25.0-mm mix).

Much research has been done on the permeability of hot mix asphalt (HMA) pavements [Ref 6, 7, 8, 9, 10]. It has been established that a number of factors influence the permeability, the two major factors being the in-place pavement density and the nominal maximum aggregate size (NMAS)

North Central Superpave Centre [6] reported that as the in-place air voids increases, permeability increases. Again as the NMAS increases, the in-place air content at which a pavement becomes excessively permeable decreases. Another factor is a mixture's gradation shape. Gradations that pass below the maximum density line (MDL), i.e. the Fuller curve tend to become excessively more permeable at lower in-place air void contents than mixtures having gradations that pass on the fine side of the MDL.

The thickness at which a pavement is placed is also another factor. As the thickness increases, the potential for permeability decreases as the chance decreases that voids will be interconnected to a sufficient length to allow water to flow. For this reason, thinner pavements may have more potential for permeability.

The National Co-operative Highway Research Program (NCHRP) [7] also reported that the in-place void content is the most significant factor impacting permeability of HMA followed by coarse aggregate ratio and VMA. Coarse aggregate ratio is defined as the ratio of coarse aggregate to fine aggregate as defined by the 4.75-mm sieve. As this ratio approached 1.0 or higher, permeability increased significantly. It was also noticed that coarse-graded mixtures generally have higher permeability values than fine-graded mixtures for a given air void.

Transit New Zealand [8] has reported that an in-place air voids of 6% or less will generally make hot mix asphalt pavements impermeable to water. NCHRP also put the figure at between 6 and 7% or less. Studies in the United States done by the National Cooperative Highway Research Program [9], however, indicated that the in-place air voids required to safeguard against any permeability problems depends upon the nominal maximum aggregate size. Specifying in-place air void contents of lower than 5, 6, 7, and 8 percent respectively for 25.0 mm, 19.0 mm, 12.5 mm and 9.5 mm mixes will also provide a safeguard against any permeable problems.

Another study done at the National Centre for Asphalt Technology, Auburn University, Alabama, USA [12] concluded that 9.5-mm and 12.5-mm nominal maximum size (NMAS) mixtures became excessively permeable at approximately 7.7 percent in-place air voids, which corresponded to a field permeability value of 100x 10^{-5} cm/sec. Mixtures having a 19.0-mm NMAS became permeable at an in-place void content of 5.5 percent, which provided a field permeability value of 120×10^{-5} cm/sec. Coarse-graded mixes having an NMAS of 25.0-mm became permeable at 4.4 percent air voids, which corresponded to a field permeability value of 150×10^{-5} cm/sec.

The results obtained by National Centre for Asphalt Technology are in agreement with those obtained by the National Cooperative Highway Research Program and both say that as the size of the Nominal Maximum Aggregate Size increases, the in-place air void to prevent permeability of water into the pavement decreases.

The dilemma with Asphalt Mix Design in Pakistan with high temperatures and serious overloading problems is that in an effort to prevent plastic flow in the asphalt pavement, the in-place air voids are being increasing whilst at the same time the Nominal Maximum Aggregate Size is also being increased. Increase in the in-place percent air voids should go with a decrease of the NMAS, **not** the other way round as is happening here. This is counter productive, for in an effort to solve one problem, another is being created.

Research done by the Asphalt Institute (e.g. MS-20 and the Transport Research Laboratory (Overseas Road Note 19) indicates that once the in-place air voids, after secondary compaction remains greater than 3 percent, plastic deformation can be avoided. The Refusal Density Design method as outlined in Overseas Road Note 19 [13] if followed can prevent plastic flow and at the same time ensure that the pavement is not permeable to ingress of water.

Table 8 below shows the causes and effects of poor impermeability values in asphalt pavement [11]. A pavement which is porous and freely allows water and air to enter will soon deteriorate.

Mix Too Permeable	
Causes	**Effects**
Low Asphalt Content	Thin asphalt films will cause aging and raveling
High voids content in design mix	Water and air can easily enter pavement, causing oxidation and disintegration
Inadequate compaction	Will result in high voids in pavement leading to water infiltration and low strength

Table 8: **Causes and Effects of Poor Impermeability**

The above suggests that the coarse gradation and the resultant permeable mix without adequate provision for drainage of the lower pavement layers is a major contributing factor to the problems.

There was a unanimous agreement between NHA, the Consultant and the Contractor that the initial cracks started from the top and propagated to the bottom, that is ‘top-down’ cracking [2, 3]. The NCHRP [10] has also published an extensive review of ‘top-down’ cracking.

8. CONCLUSIONS

- Although the Job Mix Formula (JMF) satisfied the NHA criteria in all aspects except compliance with Voids Filled with Binder (VFA). VFA is not part of NHA specification as this is difficult

to achieve with a coarse graded mix and is considered to lead to higher bitumen content with the resultant pavement more susceptible to rutting in hot climatic conditions.

- The presence of the cracking however suggest that there is a limit at which a mix that might otherwise be less prone to rutting under heavy traffic loads will be otherwise damaged by these same heavy loads under a certain combination of circumstances. Durability shall be the main criteria.
- High porosity of the mix is likely due to the larger nominal maximum aggregate size (NMAS), in addition to the low binder content. A lean mix accompanied by high porosity and heavy traffic loading will induce cracks in the pavement. Two specific characteristics i.e. higher permeability and low binder content (in relation to MS-2 criteria) of the mix have been identified as contributing to the pavement cracking. The porous mix was therefore one of the main reasons for the premature failure of the pavement.
- The design grading of both the asphalt concrete wearing course and the asphalt concrete base course have raised doubts following their performance under varying loading and weather conditions existent in Pakistan.
- In almost all developing countries, there are problems with overloading and high tire pressures. These two factors aid in the rapid deterioration of pavements. The overloading and high type pressures are indirectly the main causes of the failure as the design of the asphalt mixtures using coarser grading with less binder is an attempt to overcome their effects. However they are considered to be secondary in this case although they contribute to the rapid development of distress that may otherwise only occur over extended time period.

The outcome of premature distress witnessed on the project under discussion has raised many questions towards a durable asphalt concrete mix grading in context of cracking, rutting and at the same time to have a relatively impermeable mix. NHA has launched a comprehensive pavement research program to devise asphalt mix formulation system based on performance based testing of materials to suit local condition. To provide road research a sustainable platform, a centre of excellence namely Highway Research and Training Centre (HR&TC) is being established with collaboration with Japan International Cooperation Agency (JICA).

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